Geotechnical Engineering

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Building Science

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Geotechnical Investigation

Proposed Multi-Storey Building
455 McArthur Avenue
Ottawa, Ontario

Prepared For

Prestwick Building Corp.

January 8, 2020

Report PG5177-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Prestwick Building Corp. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 455 McArthur Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the investigation were to:

- ❏ Determine the subsoil and groundwater conditions at this site by means of boreholes.

- ❏ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

2.0 Proposed Project

Based on the available drawings, it is understood that the proposed residential building will consist of a multi-storey structure with 1 basement level and 1 sub-basement level which extends to an approximate geodetic elevation of 67.5 m. It is anticipated that the proposed development will also include asphalt-paved access lanes and parking areas around the proposed building. It is further understood that the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was conducted on November 29 and December 6, 2019 and consisted of 3 boreholes advanced to a maximum depth of 6.7 m below the existing ground surface. The borehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the boreholes are shown on Drawing PG5177-1 - Test Hole Location Plan included in Appendix 2.

Borehole BH 1 was advanced using a truck-mounted auger drill rig while boreholes BH 2 and BH 3 were completed using portable drilling equipment, both of which were operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. All samples were visually inspected and initially classified on site and subsequently placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the auger and split spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Rock samples were recovered from boreholes BH2 and BH3 using a core barrel and diamond drilling techniques. The depths at which rock core samples were recovered from the boreholes are shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Groundwater monitoring wells were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

All samples will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson. The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), consisting of the top grate of the catch basin located on the south side of McArthur Avenue with a geodetic elevation of 71.06 m. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5177-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Soil samples will be stored for a period of one month after this report is completed, unless otherwise directed.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine its concentration of sulphate and chloride along with its resistivity and pH. The laboratory test results are shown in Appendix 1 and the results are discussed in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a two storey residential building fronting onto McArthur Avenue and a single storey garage on the north end of the site. An asphalt-paved access lane is located along the eastern edge of the property and a small parking area is located directly behind the residential building. The site is bordered by McArthur Avenue to the south, commercial properties to the east and west, and a residential property to the north. The ground surface across the site is relatively level and at-grade with McArthur Avenue at approximate geodetic elevation 71.3 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of an approximate 1.1 to 2.7 m thickness of fill at ground surface or underlying the asphalt surface. The fill was generally observed to consist a brown silty sand with trace silty clay and some gravel and crushed stone.

A glacial till deposit was observed underlying the fill consisting of a loose to compact, brown sandy silt to silty clay with fragments of weathered bedrock.

Bedrock

Bedrock was encountered underlying the glacial till at approximate depths of 1.8 to 3.1 m below the existing ground surface. The bedrock encountered at borehole BH1 was augered to an approximate depth of 6.7 m. Bedrock was cored at boreholes BH2 and BH3 to approximate depths of 4.0 to 4.7 m. The bedrock consisted of a severely weathered shale and based on the RQDs of the recovered rock core, the weathered bedrock can be classified as very poor in quality.

Based on available geological mapping, the bedrock at the subject site consists of shale of the Billings formation with a drift thickness of 2 to 5 m.

4.3 Groundwater

Groundwater levels were measured in the monitoring wells installed in the boreholes on December 11, 2019. The observed groundwater levels are summarized in Table 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1	71.20	3.92	67.28	December 11, 2019
BH 2	71.30	2.37	68.93	December 11, 2019
BH 3	71.38	3.50	67.88	December 11, 2019

It should be noted that the groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 3 to 4 m below ground surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed multi-storey building. It is expected that the proposed building will be constructed using conventional shallow footings bearing on the undisturbed, weathered shale bedrock.

Removal of the weathered bedrock will be required to complete the excavation for the proposed building. This is discussed further in Section 5.2.

Expansive shale bedrock could be present on site. Precautions should be provided during construction to reduce the risks associated with the potentially heaving shale bedrock. This is discussed further in Section 6.8.

The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, asphalt and fill, containing deleterious or organic materials, should be stripped from under any building, paved areas, pipe bedding and other settlement sensitive structures. Under paved areas, existing construction remnants, such as foundation walls, pipe ducts, etc., should be excavated to a minimum depth of 1 m below final grade.

Bedrock Removal

In consideration of the very poor quality of the weathered bedrock encountered within the proposed depth of excavation, it is anticipated that bedrock removal will be possible using hoe-ramming and conventional excavation techniques.

Vibration Considerations

Construction operations could be the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain a cooperative environment with the residents.

The following construction equipments could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations caused by construction operations could be the cause or the source of detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the permissible vibrations, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Fill used for grading beneath the proposed building should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Bearing Resistance Values

Footings placed directly on the undisturbed, weathered bedrock can be designed using a bearing resistance value at SLS of **500 kPA** and a factored bearing resistance value at ULS of **1,000 kPa**.

An undisturbed bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed material, whether in-situ or not, have been removed, under dry conditions, prior to the placement of concrete for the footings.

Footings designed using the above-noted bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a weathered bedrock above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**. If a higher seismic site class is required (Class A or B), a site specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012.

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the OBC 2012 for a full discussion of the earthquake design requirements.

5.5 Sub-Basement Floor Slab

Based on the anticipated depth of the proposed basement level, the bearing medium for the sub-basement floor slab will consist of weathered bedrock.

It is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its SPMDD.

A sub-floor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear crushed stone backfill under the lower basement floor. This is discussed further in Subsection 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Pavement Structure

Car only parking areas and access lanes are anticipated at this site. The proposed pavement structures are presented in Tables 2 and 3.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
50	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
300	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 3 - Recommended Pavement Structure - Access Lanes

Thickness (mm)	Material Description
40	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
50	Binder Course - HL-8 or Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
450	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed building. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Where insufficient room is available for exterior backfill, it is suggested that the composite drainage system (such as Delta Drain 6000 or equivalent) be secured against the shoring system extending to a series of drainage sleeve inlets through the building foundation wall. The drainage sleeves should be at lease 150 mm diameter and be spaced 3 m along the perimeter foundation walls. An interior perimeter drainage pipe should be placed along the building perimeter along with the sub-slabdrainage system. The perimeter drainage pipe and sub-slab drainage system should direct water to sump pit(s) within the lower garage area.

Sub-slab Drainage

Sub-slab drainage will be required to control water infiltration. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximate 6 m centres. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Where sufficient space is available, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose. A waterproofing system should be provided for any elevator pits (pit bottom and walls).

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover should be provided for adequate frost protection for heated structures.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials and weathered bedrock should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. Based on the depth of the proposed structure and the proximity to property lines, it is anticipated that a temporary shoring system will be required to support the excavation on the south, east and west sides.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring is anticipated to be required to support the overburden soils and the weathered bedrock. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration a full hydrostatic condition which can occur during significant precipitation events.

The temporary shoring system may consist of a soldier pile and lagging system which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of rock bolts or extending the piles into the bedrock through pre-augered holes, if a soldier pile and lagging system is the preferred method.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 4 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	21
Submerged Unit Weight (γ), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 98% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.5 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material standard Proctor maximum dry density.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Impacts on Neighbouring Properties

Based on our observations, localized groundwater lowering may be required under short-term conditions due to construction of the proposed building. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site due to the minimal temporary groundwater lowering.

Further, due to the presence of shallow glacial and/or bedrock at, and in the vicinity of, the subject site, the neighbouring structures are expected to be founded on the glacial till or bedrock. Therefore, no issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsoil conditions at this site mostly consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters, tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of an aggressive to very aggressive corrosive environment.

6.8 Protection of Potentially Expansive Bedrock

The presence of potentially expansive shale is anticipated to be encountered at the subject site. To reduce the long term deterioration of the shale, exposure of the bedrock surface to oxygen should be kept as low as possible. The bedrock surface within the proposed building footprint should be protected from excessive dewatering and exposure to ambient air. Therefore, a 50 mm thick concrete mud slab, consisting of 15 MPa lean concrete, should be placed on the exposed bedrock surface within a 48 hour period of being exposed.

Another option for protecting the shale from deterioration is placing granular fill over the exposed surface within a 48 hour period after exposure. Preventing the dewatering of the shale bedrock will also prevent the rapid deterioration and expansion of the shale bedrock.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials used.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine its suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than the Prestwick Building Corp. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Kevin A. Pickard, EIT.

Scott S. Dennis, P.Eng.

Report Distribution

- ❑ Prestwick Building Corp. (e-mail copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
455 McArthur Avenue
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5177

REMARKS

HOLE NO. BH 1

BORINGS BY CME 55 Power Auger

DATE 2019 November 29

SOIL DESCRIPTION	Layer bottom depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	71.20
Asphaltic concrete	0.04						
FILL: Dark brown silty sand, some gravel, trace clay	1.07	AU	1				
		SS	2	88	9	1	70.20
GLACIAL TILL: Dark brown sandy silt with shale fragments	2.29	SS	3	100	18	2	69.20
GLACIAL TILL: Brown silty clay with shale fragments, trace sand	3.05	SS	4	100	19	3	68.20
BEDROCK: Very poor quality, black shale	6.70	SS	5	100	76		
		SS	6	100	50+	4	67.20
		SS	7	67	50+		
		SS	8	0	50+	5	66.20
		SS	9	0	50+	6	65.20
End of Borehole							
(GWL @ 3.92m - Dec. 6, 2019)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

Monitoring Well Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
455 McArthur Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Portable Drill

DATE 2019 December 6

FILE NO. PG5177

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	71.30
FILL: Brown silty sand, trace topsoil and asphalt 0.30	AU	1				
FILL: Brown silty sand, trace gravel 0.60	SS	2	71			
FILL: Brown silty sand, some clay, trace gravel	SS	3	54		1	70.30
	SS	4	83		2	69.30
2.74	SS	5	96			
BEDROCK: Very poor quality, black shale	RC	1	64	0	3	68.30
4.72	RC	2	46	0	4	67.30

End of Borehole

(GWL @ 2.37m - Dec. 11, 2019)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

Monitoring Well Construction

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
455 McArthur Avenue
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5177**

REMARKS

HOLE NO. **BH 3**

BORINGS BY Portable Drill

DATE 2019 November 29

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20, 40, 60, 80)	Monitoring Well Construction
GROUND SURFACE						0	71.38		
FILL: Brown silty sand, trace topsoil, organics (0.30)		AU	1						
FILL: Brown silty sand, trace gravel		SS	2	92					
		SS	3	50		1	70.38		
(1.52) **GLACIAL TILL:** Brown silty sand with shale fragments (1.83)		SS	4	100					
BEDROCK: Very poor quality, black shale		RC	1	35	0	2	69.38		
		RC	2	90	0	3	68.38		
(4.57) End of Borehole						4	67.38		

(GWL @ 3.50m - Dec. 11, 2019)

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Compactness Condition	'N' Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

Consistency	Undrained Shear Strength (kPa)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	ROCK QUALITY
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC%	-	Natural water content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic Limit, % (water content above which soil behaves plastically)
PI	-	Plasticity Index, % (difference between LL and PL)
Dxx	-	Grain size at which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu	-	Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: 1 < Cc < 3 and Cu > 4
Well-graded sands have: 1 < Cc < 3 and Cu > 6
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

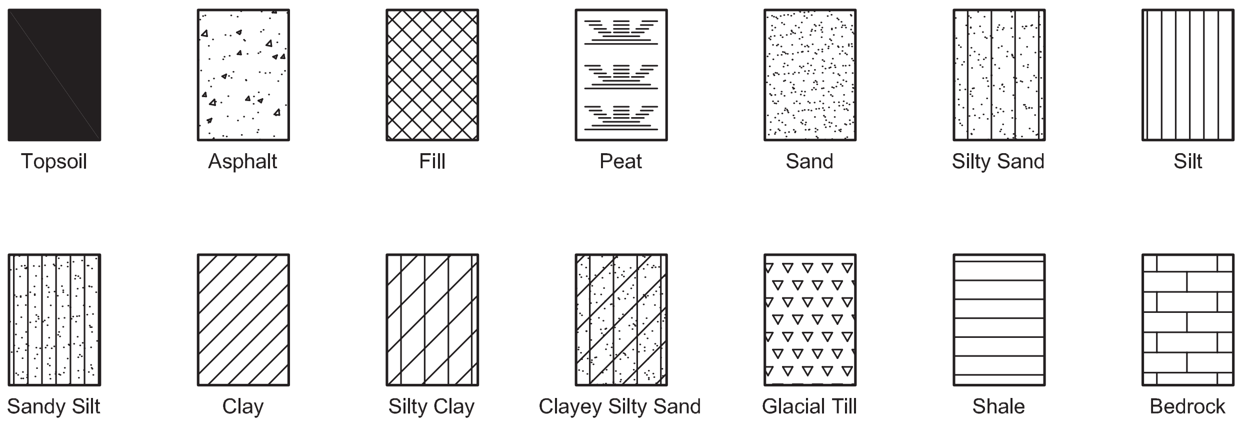
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

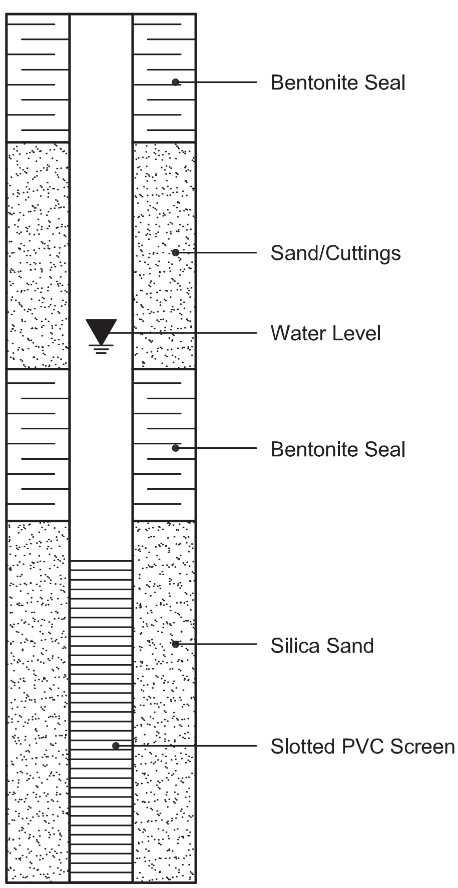
SYMBOLS AND TERMS (continued)

STRATA PLOT

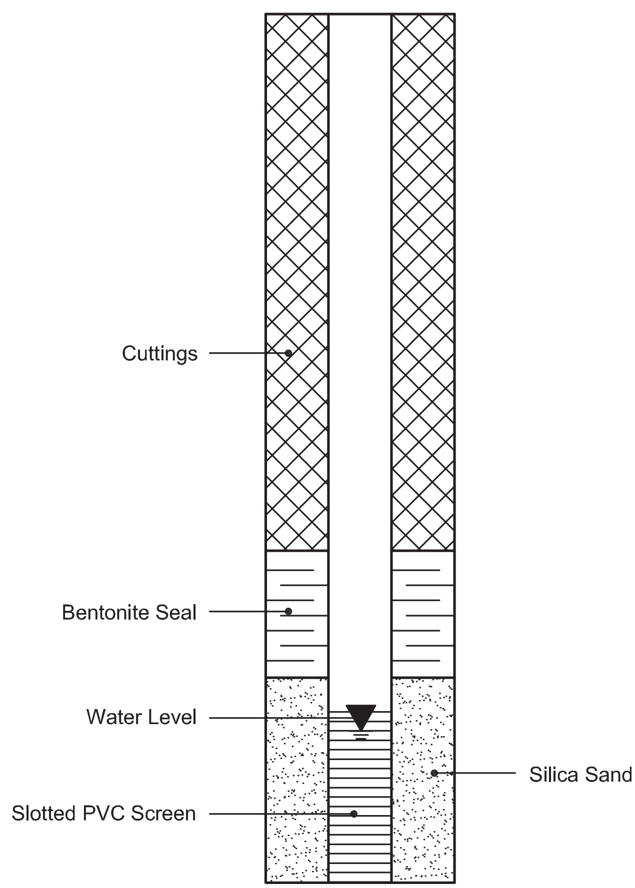


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 27097

Report Date: 04-Dec-2019

Order Date: 29-Nov-2019

Project Description: PG5177

	Client ID:	BH1-SS3	-	-	-
	Sample Date:	29-Nov-19 10:00	-	-	-
	Sample ID:	1948646-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	80.3	-	-	-
General Inorganics					
pH	0.05 pH Units	6.68	-	-	-
Resistivity	0.10 Ohm.m	58.2	-	-	-
Anions					
Chloride	5 ug/g dry	61	-	-	-
Sulphate	5 ug/g dry	14	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5177-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

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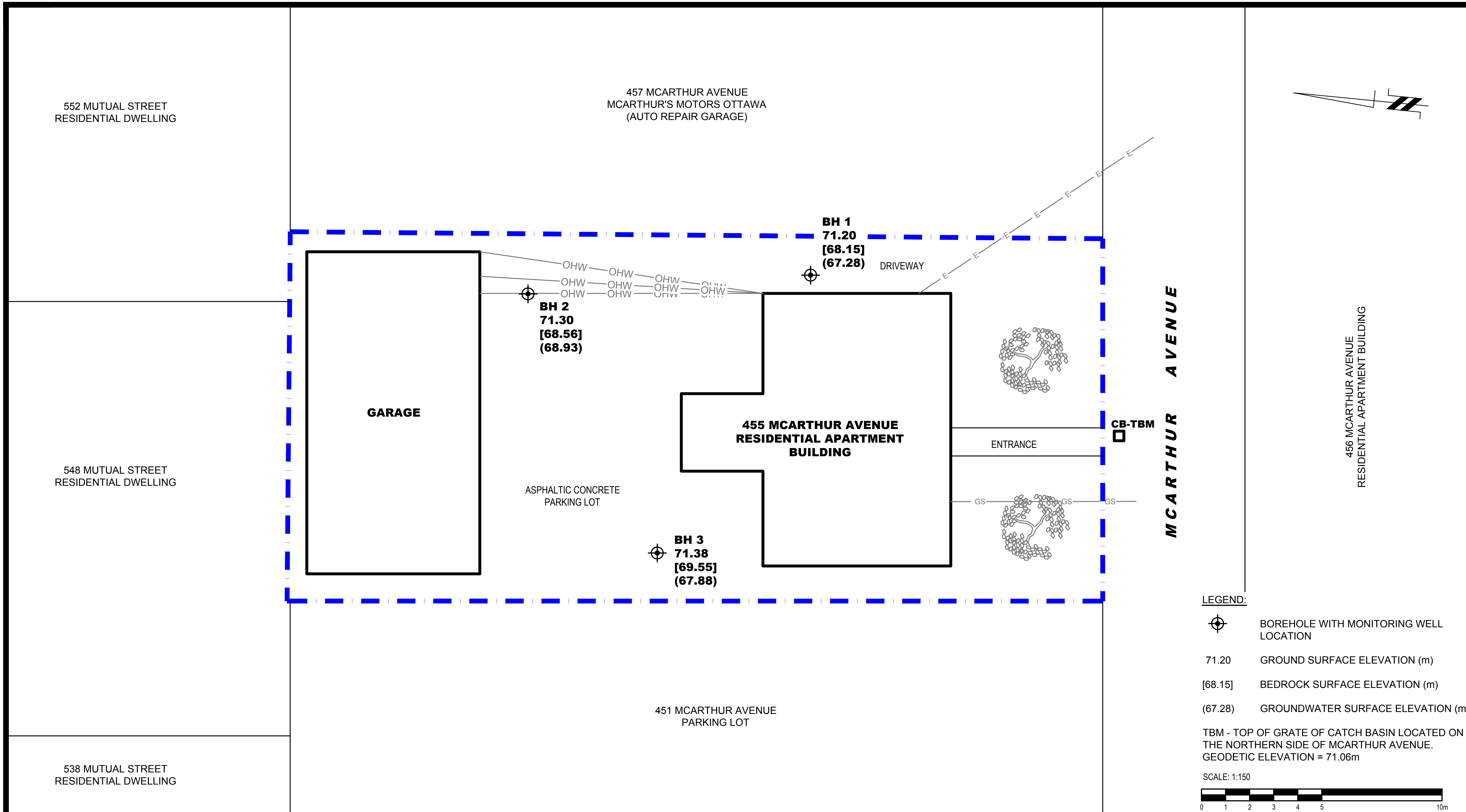

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NO.	REVISIONS	DATE	INITIAL

PRESWICK BUILDING CORPORATION

GEOTECHNICAL INVESTIGATION
455 MCARTHUR AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:150	Date: 01/2020
Drawn by: NFRV	Report No.: PG5177-1
Checked by: KP	Dwg. No.: **PG5177-1**
Approved by: DJG	Revision No.:

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